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HYDRAULIC MODEL STUDIES OF CEBADA AND PURISIMA WASTEWAY STRUCTURES

by

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INTRODUCTION AND ACKNOWLEDGMENTS

This report contains the results of model tests of the Cebada and Purisima Wasteway structures conducted in the Hydrodynamics Laboratory of the California Institute of Technology, Pasadena, California, during the period from June to October, 1949. The study was carried out under a cooperative agreement between the Soil Conservation Service of the United States Department of Agriculture and the California Institute of Technology.

The two structures are part of a flood control project designed to protect the valley farmlands northeast of Lompoc, California. The original design of the wasteway structures was made by Mr. C. C. Rich of the Soil Conservation Service, Santa Barbara, California, under the general supervision of the District Conservationist, Mr. Nelson P. Rutherford, as part of the complete work plan for the Purisima and Cebada Sub-Watersheds.

Mr. Rich also was the technical representative of the Soil Conservation Service on the model studies and assisted materially by guiding the work through discussions during frequent visits to the laboratory. Much credit is due the directors of the Lompoc Soil Conservation District for the support they have given the work. Two of the directors actually visited the laboratory and witnessed some of the experiments.

The experimental work was done by Messrs. J. T. O'Brien and Max Kreston under the direction of Dr. Vito A. Vanoni. Mechanical, photographic and other services were provided by the regular staff of the Hydrodynamics Laboratory.

A. Statement of the Problems

The Cebada and the Purisima watersheds are located in the Lompoc Soil Conservation District about three miles northeast of Lompoc, California. The streams in those two watersheds empty into the Santa Ynez River about ten miles from its mouth after emerging from a well entrenched channel and flowing across about one mile of fertile valley bottom land. Because the stream grades flatten on the valley floor, sediment is deposited which fills up any artificial channels that may be built to carry the flows to the river. The result is that the valley lands and crops suffer flood damage at frequent intervals.

To correct this situation, the Soil Conservation Service designed a system of reinforced concrete flood control channels to carry flows from these two streams across the valley bottom to the river. Cost considerations eliminated the possibility of constructing the channels large enough to carry flows from major storms and dictated that only floods of more frequent occurrence could be carried. In order for such a system to function, it was necessary to divert or waste that portion of the flow in excess of the capacity of the concrete channel to the river. The plan prepared by the Soil Conservation Service called for diverting the excess flow into the existing natural flood channels by means of a side channel wasteway structure.

The problem of the Laboratory was to study by hydraulic model the performance of the wasteway structures for the two channels and to recommend designs.

The overall plan for the flood control system for the Cebada and Purisima watersheds also calls for channel improvement and control works such as dams and bank revetment, and changes in land use. These should reduce the amount of sediment and therefore the problem of transporting it

to the river in the concrete channels. However, because there will always be some sediment in the flows, it was necessary to design the wasteway structures so they would be self-cleaning.

B. Field Data

Fig. 1 and the frontispiece are aerial photographs of the Purisima and Cebada watersheds showing the land use and the main features of the flood control system. The two wasteway structures may be seen in Fig. 1 at the mouths of the respective valleys just south of the Camp Cook road. It will be noted that these structures are located at the intersection with the original natural streams just a short distance downstream from the intake to the concrete channels. The two lined channels finally come together into a single channel about one mile from the river. The land protected by the channel system lies to the south and west of the wasteway structures.

Estimates of runoff for the two watersheds at the sites of the respective wasteway structures are given in the following table:

Frequency	Flow in cfs	
years	Cebada	Purisima
10	300	265
50	600	635

The final plan prepared by the Soil Conservation Service proposed to carry the 10-year frequency flow to the river in concrete channels and to waste the excess flow at the wasteways. For instance, for the Cebada channel all flows up to 350 cfs will be carried to the river in the concrete channel. But when the flow increases to 400 cfs, then 50 cfs will be wasted. These specifications had to be modified slightly because it is impossible to design a wasteway structure to perform strictly according to them. This point will be discussed further in a later section.

As indicated previously in this report, the flood control plan for the Cebada and Purisima watersheds included dams in the entrenched channels as well as bank revetment and modification of land use. This work will reduce the sediment load and the severity of the problem of transporting the sediment load in the concrete channel. The dams, several of which have already been built, are immediately effective in holding back sediment since they have appreciable storage capacity which will not be filled for several seasons. The dams will also flatten and stabilize the grades, thus reducing the sediment carrying capacity of the streams and the amount of material carried into the concrete channels.

A few samples were taken from the beds of the streams. These show that the material in the Cebada stream has an average grain size of about .25 mm. and that in the Purisima stream the average size is .15 mm.

C. Design of the Models

A model basin 10 ft. wide, 2 ft. deep and 70 ft. long was available for the study of these structures. This basin can circulate flows in excess of 2 cfs and is equipped with Venturi meters for measuring the flow and other apparatus suitable for this type of study. The two wasteway structures have approximately the same capacity and, therefore, are also approximately the same size. After a study of the two designs, a length scale of 1:12 was selected for the models. This allowed adequate length in the model basin to represent inlet and discharge conditions. Fig. 4 is a photograph of the model of the Cebada wasteway structure in the test basin. The model scales based on the Froude similarity law can be summarized as follows:

Quantity	Scale ratio
length	1:12
time	1:3.46
velocity	1:3.46
discharge	1:499
Manning roughness	1:1.52

It can be seen immediately that with an available flow of 2 cfs there is ample discharge to represent the maximum flow expected in the structures.

From the prototype data presented in section B and in Figs. 2 and 3, it is determined that the minimum velocity in the structures is about 5 fps when the hydraulic radius is about 2.5 ft. Using the above scale ratios, the corresponding hydraulic radius and velocity in the models are 0.22 ft. and 1.4 fps, respectively. These conditions give a Reynolds number in the model in excess of 100,000, which is well into the turbulent flow range. In calculating this Reynolds number, the length parameter has been taken as 4 times the hydraulic radius in order to make the Reynolds number comparable to those used in pipe flow calculations. The fact that the model velocities are appreciable also reduces any possible surface tension effects to very small proportions and no difficulties should be expected from this source.

Estimates by the Soil Conservation Service indicated that the Manning roughness values for the concrete channels and the earth channels would be 0.015 and 0.035 respectively. Using the roughness scale listed above, the corresponding roughness values for the model are 0.0099 and 0.0230 respectively. Roughness value of 0.0099 can be obtained by using painted wood which was the material used in constructing the models. The approach sections representing the earth channels were made up of a brass flume with transition sections made either of smooth concrete or metal and wood. The slope of these approach sections was adjusted to give the proper approach velocity and depth and no attempt was made to reproduce the prototype roughness. Since considerable change in velocity takes place in these structures, no difficulty was anticipated because of any possible minor deviations from the roughness of the model.

No attempt was made to simulate the sediment load in the streams although the presence of sediment in the prototype was recognized. The models of the earth channels were made with fixed beds as indicated above. During the course of the studies care was taken to keep the velocity in the wasteway structures as high as possible in order to keep the sediment transporting capacity as high as possible and to minimize any possible sedimentation problems that may arise.

D. <u>Description of the Designs and their Modifications</u>

1. (a) <u>Cebada Basic Design</u>

Figure 2 shows the design for the Cebada Wasteway structure as made by the Engineering section of the Soil Conservation Service, Santa Barbara, California. It represents their solution to the problem as stated and discussed in Section A. This is referred to as the "basic design" in the text that follows. The side walls of the model are all 10 ft. in height. The concrete wasteway structure was designed so that the velocities in it would be consistently higher than those in the natural channels upstream. This was done in an attempt to make the sediment transporting capacity of the lined channels as high as possible.

Due to commitments of right of way and cost, it is possible to change only slightly the detailed widths and slopes of the channel from those in Fig. 2. The length of the side weir is fixed at 70 ft. and it is specified that the best distribution of flow possible be obtained across this weir. The factor of roughness, n, in the Manning formula used in the prototype design is .015 for the concrete channels and .035 for the earthen channels.

Table D-1 below presents values of the storm flows as calculated by the hydrologists, together with the specified spill over the side weir and flows under the bulkhead which are to be carried to the river in the concrete channel.

Tablo D-1
Specified Inflows and Spillovers for Cebada Wasteway

Storm froq. (years)	Dischargo (cfs)		
	Total	over sideweir	under bulkhead
10	300	0	300
50	600	250	350

In addition, it was specified that the behavior of the wasteway be investigated for an extreme overload flow of about 800 cfs.

It will be noted that according to the above specifications, side spill should occur while the through channel to the river is still not carrying its full capacity. It would be desirable for no spill to occur until the capacity of the through channel is reached, but the characteristics of the structure used are such that this is not possible.

(b) Modification No. 1 to Cebada

In order to study the effect on the performance of the side weir of a reduction in the velocity of approach to the weir, the transition from the trapezoidal to the rectangular section was modified as shown in Fig. 2-b. The slope of the side walls was not altered but the drop in the transition was reduced from 2.8 to 1.2 ft. and the widths other than at the entrance and exit were slightly increased.

(c) Modification No. 2 to Cebada

In this modification all of the details of the basic design were retained down to the upstream end of the side weir. The side weir and bulkhead were altered as shown in Fig. 2-c. The weir was pitched uniformly so that the upstream and downstream ends were respectively 4.2 and 5.6 ft. abovo the floor. The bulkhead was at station 88 + 68 as in the basic design but it was lowered so the distance from the floor to the lip was 3.4 ft. and the upstream face of the lip was rounded in a circular arc. The arc was of 1/2 ft. radius and tangent to the upstream face and perpendicular

to the downstream face. The cross section of the weir was as in the basic design. The apron was 3.7 ft. above the floor and parallel to it for its full length and the wing walls at the upstream and downstream ends of the apron remained as in the basic design.

2. (a) Purisima Basic Design

The details of the basic design, as made by the engineering office of the Soil Conservation Service, Santa Barbara, are presented in Fig.3-a. The design represents their solution to the problem as stated and discussed in Section A.

The main design considerations are summed up in the letter from the Soil Conservation Service, which transmitted the design drawings to the laboratory. It reads in part:

> "....This design has been made along the same assumptions as the Cebada Wasteway structure.... The principal difference lies in the fact that the Purisima Channel is picked up at the outlet end of a culvert instead of from open channel flow (as in Cebada). The culvert forms an angle with the channel downstream from the culvert of 12°. This angle should be turned within 40 feet of the culvert outlet either in a smooth curve or an angle bend, whichever proves the most effective from model studies...."

As with the Cebada design, it is intended that the velocities in the Purisima wasteway be kept high so that sediment will not deposit in the flume.

Again, as with the Cebada design, it was possible to vary the design widths and slopes only slightly due to right of way commitments and cost estimates. The length of the side weir is fixed at 70 ft. and it was desired that the best flow distribution possible be obtained across it.

Table D-2 below presents values of the storm flows as calculated by the hydrologists together with the specified spill over the side weir for these flows.

Table D-2

Specified Inflows and Spillovers for the Purisima Wasteway

Storm freq. (years)	Discharge (cfs)		
	Total	over sideweir	under bulkhead
10	265	0	265
50	635	320	315

(b) Modification No. 1 to Purisima

In this modification, the turn of 12° at the culvert exit shown in the basic design was changed to two turns, one of 6° at the culvert exit and one of 6° - 20' at a point downstream 15 ft. The channel widths were not changed so there remained a contraction of the channel between the culvert exit (station 59 + 95) and station 59 + 60 from 7.6 ft. to 7.0 ft. A third turn of 0° - 20' was made at station 57 + 00, which is 20 ft. downstream from the bulkhead, to bring the alignment which was displaced by the two turns upstream back to the original channel alignment. The details of the modification are shown in Fig. 3-b.

(c) Modification No. 2 to the Purisima Design

The side weir of the basic design was parallel to the bottom of the flume, as shown in Fig. 3-c. In modification No. 2 the weir was pitched such that its upstream end and downstream end were respectively 3.3 and 6.0 ft. above the floor bottom. The cross section of the weir was kept flat crested and 1/2 ft. wide, as in the basic design. The apron was kept parallel to the bottom of the flume as in the basic design, but its elevation was lowered to 2.7 ft. above the floor, or 1/2 ft. below the lowest point on the weir crest. The height of the wing walls was not changed and they were located at the upstream and downstream ends of the apron, as in the basic design.

E. Apparatus and Procedure

A propeller pump operated by an electric motor and housed in a pump house at the east (downstream) end of the flume, as shown in Fig. 4, furnished the water to the models. The flow from the pump is through a 12-in. diameter pipe and the rate of discharge in the pipe is measured by Venturi meters. Water enters the model through a rectangular approach channel 18 in. wide which receives water from the stilling box shown at the upper left of Fig. 4.

Each flume contains a side spillway for which the rate of discharge was determined by making a volumetric measurement using the rectangular sheet metal tank shown in the right center position of Fig. 4. The tank is fixed and calibrated in reference to the attached point gage. The flow from the side spillways was conducted into the tank by means of a sheet metal swing chute and the time of filling was obtained by use of a stop watch.

Water depths were measured with a point gage fastened to a carriage which permitted movement of the point gage in a horizontal plane, both longitudinally and laterally.

Cameras were used to record water surface profiles and configurations, and the direction of the surface flow was obtained by taking long exposure photographs of floats on the water surface. The photographs taken included both still and moving pictures and were all in black and white.

Visual observations of all features of the performance of the structures were made. These included the configuration of the flow over considerable time periods, the investigation in particular parts of the structures of the effect of proposed modifications, and the immediate effect of the addition of a small amount of sediment to the flow. Particular attention was given to the study of the flow patterns by the insertion of dyes and particles of paper in the flow. Generally, many of the dynamic

effects which showed some change over a period of time were observed visually to augment the data collected by direct measurement and photographs.

F. Summary of Results of Tests of Cebada

1. Basic Design

With the 50-year storm flow of 600 cfs diamond pattern waves occur in the channel downstream from the transition from the earthen to the rectangular concrete channel and the concentration of the bulk of the spillover is in the lower 30 percent of the 70-ft. long side weir, as shown in Figs. 5 and 6. The profile through the transition shows a marked drawdown to a minimum depth at about station 89 + 88. The depths from this station to the upstream edge of the weir are all below critical, while the reverse is true for those between the downstream edge of the weir and the bulkhead. Since the distribution of the flow over the side weir was not measured, the velocities in this stretch are not known. However, observation in the model with a flow of 600 cfs show that a very shallow wave cannot be propagated upstream in this reach, therefore it is concluded that the stream velocities are greater than critical. Under these conditions of flow the bulkhead has only a local effect which for this flow is noticeable in only the downstream 20 ft. of the weir.

In the weir section none of the sharp breaks in the water surface, as calculated and presented in Fig. 6-c, were observed, but rather all of the transitions including the large undular jump at about station 89 + 00 are gradual. The calculated profile was determined by step integration of the standard backwater equation to which had been added a term to account for the side spill.

Reference to Fig. 7-b will show that the downstream wing wall is overtopped by about 1 ft., and that the depths of water along the field edge of the apron vary from 0 ft. at the east end to 1 1/2 ft. at the west (downstream) end.

Of particular interest is the relation between the total inflow, Q_t, and that which passes under the bulkhead, Q_c, since the difference, Q_{sw}, must discharge over the weir and be wasted on the adjoining field. The ratio of Q_c/Q_t for the 50-year flow is specified as 350/600 = .58 but as measured in the model this ratio is 375/600 = .63. That is, 350 cfs is desired in the lower channel but 375 cfs is measured in the model as shown in Fig. 8. Thus the structure built according to the basic design does not permit enough spill over the side weir. This condition would be advantageous to the immediate landowner who stands to receive the additional increment but it would overload the lower concrete channel and might cause overflow and considerable damage.

As shown in Fig. 6, for the flow Q_c, of 375 cfs, the measured depth at a point 10 ft. downstream from the bulkhead is about 2.7 ft., while that calculated is 4.2 ft. The difference is due to the decided contraction of the flow as it passes under the bulkhead and is thereby accelerated.

As indicated in Section A of this report, the flow into the Cobada wasteway for a 10-year storm is 350 cfs. The design specifications require that this be the maximum flow for which there is no side spill. The discharge curve in Fig. 8 shows that with the basic design as the stage rises, a flow of 420 cfs is reached before side spill begins and that when the stage is falling, side spill stops when a flow of 320 cfs is reached. This means that on the rising stage the lower channel will be overloaded by 70 cfs and that on the falling stage water would be spilling when the channel below the bulkhead was not running full.

The hysteresis effect, or the difference between the distribution of flow under the bulkhead and over the weir on rising and falling stages, is due to the contraction of the flow when the bulkhead is functioning. As soon as the stage rises high enough to strike the face of the bulkhead, the

flow is forced to contract and the water level upstream from bhe bulkhead must rise in order to produce enough head to discharge the flow. If the flow rate is now reduced slightly, the head or the water surface elevation at the bulkhead will be reduced but some head will still be required to force the flow under the bulkhead so the water will stay high and spill-over will occur. The water surface will spring free of the bulkhead only when the flow rate is reduced to the point where no head is required to force the water under the bulkhead.

The profile in Fig. 6-b shows the back-up effected by the bulkhead when the total in-flow is 360 cfs on a falling stage and as can be seen, the water surface has risen above the weir crest, and therefore, some of the flow is spilled over the side. During a rising stage, all of this water and more would pass under the bulkhead and the spill of course would be zero.

In Fig. 6-a is shown the profile for a total flow of 300 cfs which is the maximum computed flow for a condition of no side spill. The water level is considerably below the bottom of the bulkhead and the lip of the weir. Due to acceleration through the transition section, uniform flow does not exist between it and the bulkhead and consequently uniform flow formulas cannot be used to determine the water depth under the bulkhead. The flow in the channel below the bulkhead is decelerating and while the channel slope downstream from station 88 + 38 increases from .003 to .009, it is likely that the flow will reach normal depth for the slope of .009 before the junction with the Purisima channel is reached approximately 3,000 ft. downstream. The drawdown beginning at station 88 + 48 is due to the effect of terminating the model at station 88 + 38 and does not represent a prototype condition.

The water surface profiles for flows of 360, 450, and 815 cfs are similar to those for the 50-year storm flow as the curves in Fig. 6

indicate. All flows show an acceleration through the transition, a gradual deceleration between the exit from the transition to near the upstream end of the side weir, an acceleration through the first part of the side weir, an undular hydraulic jump near the center of the weir, and a deceleration as the flow nears and strikes the bulkhead. The flow of 815 cfs is a super flow and therefore considerably overtaxes the capacity of the flume, especially at the bulkhead. Such a flow may overtop the earthen approach channel and therefore part may bypass the wasteway. These flows exhibited the characteristic drawdown or contraction under and downstream from the bulkhead.

Figure 7 indicates that in the basic design the bulk of the spill-over is concentrated in the downstream 30 feet of the side weir and that the flow along the field edge at the apron is distributed in the same manner. The depths at the downstream edge of the apron are shown as 0.4 ft. and 3.0 ft. for side weir discharges of 300 and 410 cfs respectively.

The discharge curve in Fig. 8 indicates that the basic design consistently permits about 25 cfs more flow through the bulkhead and into the lower channel than required by the design specifications. The hysteresis loop in the curve between 300 and 420 cfs total flow has already been discussed.

2. <u>Modification No. 1</u>

The details of modification No. 1 are shown in Fig. 2 and discussed in Section D. The modification involved a reduction of the net drop in the transition section from 2.8 ft. to 1.2 ft. The slope of the side walls was left as in the basic design but the channel widths, exclusive of the exit and entrance, were slightly increased. This modification was made to permit the study of reduced velocity of approach on the distribution of the flow along the crest of the flat side weir principally for the 50-year storm flow of 600 cfs. Photographs of its performance are shown in Fig.9.

For the 50-year storm the modification causes the velocity at the transition exit, station 89 + 98, to be reduced from 13.6 fps to 11.4 fps as computed from the data in Fig. 10-a. However, in spite of this reduction, the velocities downstream are nearly the same as those with the basic transition, and the profiles for the basic and modified designs practically coincide.

Since the profile along the weir section is not altered by the modification, neither the discharge over the side weir nor its distribution is changed, but they remain the same as for the basic design as shown in Figs. 7 and 10. The depth at the face of the bulkhead is not altered, therefore the discharge under it also remains unchanged.

Figure 10-c presents the results of tests (run No. 19) of modification No. 1 with side weir and bulkhead lowered to 3.2 ft. above the floor at all points. Also shown in the same figure is the profile obtained from run No. 15. In this run the transition is as in Modification 1, but the lower channel is completely closed by a bulkhead so that the total flow is forced to go over the side weir.

The profiles in Fig. 10-c show that notwithstanding the very dissimilar types of approach to and exit from the weir, the profiles between the upstream end of the weir and a point 50 ft. downstream therefrom are practically identical. The profiles downstream from the latter point are, of course, different to account for the difference of 300 cfs in the discharge over the side weir. Thus it is seen that the behavior over most of the weir section is insensititive to approach and outlet conditions.

The data in Fig. 10-c indicate that the weir itself is a control independent of the bulkhead and that for a particular crest elevation a particular drawdown curve is defined by the flow over the weir. Also it is apparent that the effect of the bulkhead with these super-critical

velocities is so localized that even when it shuts off the flow completely it is unable to act upstream from about the mid-point of the side weir.

In Figs. 10-a and 10-b are presented the profiles for the 10-year storm flow of 300 cfs with the basic design and Modification No. 1 thereto. The approach of the flow to the transition is about the same for both designs since the depths 10 ft. upstream from the entrance are about 3 ft. in both cases. However, the acceleration through the transition is much greater with the basic design so that the depths 10 ft. downstream from the transition exit are 2 1/2 and 4 ft., respectively, for the basic and modified designs. The lower velocities in the rectangular channel with the modified design are not desirable from the standpoint of sediment transporting capacity.

Figure 10-b shows the profile for a total flow of 45 cfs with the side weir closed and the lip of the bulkhead set 3.4 ft. above the floor at station 88 + 68. When the side weir is closed the stage rises until it is high enough at the bulkhead face to force the total inflow under it. The approach velocities are very low so that the backwater effect of the bulkhead carries a considerable distance upstream compared with its very local effect when the side weir is open. The performance of the side weir for the 50-year storm was not materially improved by Modification No. 1. For the 10-year storm flow the velocities in the rectangular channel are noticeably slower with Modification No. 1 than with the basic design. Therefore, study of this modification was discontinued after work with the 10-year and 50-year storms was completed.

3. Modification No. 2

Since Modification No. 1 was not satisfactory, the experiment was continued using the 2.8 ft. transition drop as in the basic design since this produced a high acceleration in the transition and downstream there-

from which is desirable to keep the sediment moving. However, the basic design gave an unsatisfactory distribution of the discharge over the side weir and also permitted too much flow to go under the bulkhead. Therefore, Modification No. 2 was made to remedy those conditions, first by pitching the side weir and second, by lowering the bulkhead and by changing its profile, all as shown in Fig. 2-c. The pitch of the weir and the elevation and profile of the bulkhead were all arrived at after considerable experimentation, but except for some data on the coefficient of discharge for the bulkhead contained in Appendix No. 1, only the final results are presented.

For the 50-year flood, the profiles upstream from the weir did not differ noticeably from those with the basic design as a comparison of Figs. 6 and 12 indicates. However, along the weir and apron, the distribution of the flow was improved by the modification as shown in Figs. 11, 12 & 13, although the bulk of the flow is still in the downstream part of the weir. A steeper pitch is necessary to distribute the side spill evenly over the weir length; however, due to the uniform pitch of the weir and the variable backwater effect of the bulkhead, one pitch will give optimum results at only one flow condition. It was decided to accept the unbalance of the spill over the weir for the 50-year flood if a nearly uniform flow distribution could be obtained for the more frequent storm flows of 400 to 500 cfs.

At the field edge of the apron between stations 89 + 10 and 88 + 78, velocities of about 8 fps exist based on data contained in Fig. 13. The downstream wing wall is overtopped by about 1/2 ft. but the depth of flow along the upstream wing wall is only about 1/10 ft. and it is, therefore, hydraulically unnecessary.

For the 50-year storm, a bulkhead set 3.4 ft. off the floor with the upstream lip rounded in a 1/2 ft. radius circular curve, as shown in Fig. 2-c, proved satisfactory since the measured discharge through it is 360 cfs or only 10 cfs greater than that specified for the lined channel. However, since the whole of the 10-year storm flow must pass under the bulkhead, the bulkhead setting had to be checked with that flow.

Figure 8 indicates that with the 10-year storm flow, the modified design will not permit spill until a flow of 360 cfs is reached on the rising stage and spilling will stop when a flow of about 300 cfs is reached on the falling stage. The 360 cfs is more than the specified flow but since the downstream channel is designed to carry nearly that amount with the 50-year storm, it was decided it would be safe to load it to the 360 cfs during the 10-year storm also. Note in Fig. 12-a that although the stage at the bulkhead is 2 ft. above the lip of the bulkhead, there is no spill over the side weir due to the support given to the decelerating flow by the pitched weir. Note, also, in this figure that a hydraulic jump occurs at about station 89 + 55, nearly 10 ft. upstream from the upstream end of the weir. The Froude numbers ($F = V/\sqrt{gd}$) 5 ft. upstream and downstream from this station are respectively 1.7 and 0.7.

The profile for a flow of 360 cfs on a rising stage is shown in Fig. 12-b. The velocities are well above critical, F = 1.6, at station 89 + 38, and the elevation of the water surface is nearly that of the bulkhead lip.

With Modification No. 2, there remains a hysteresis effect, as may be seen in Fig. 8; however, the lag is only 60 cfs as compared with 100 cfs which existed with the basic design. Most of the reduction is due to the decrease in the contraction of the flow through the bulkhead by the rounding of its upstream lip.

Figure 12 gives the water surface profiles for storm flows of 360, 450, and 815 cfs. The flow of 360 cfs is on a falling stage and its profile is similar to that for 300 cfs flow on the falling stage. The hydraulic jump occurs at about the same station for both flows and the velocities downstream from the jump are still high and can move a considerable sediment load. The shape of the water surface profile with a flow of 815 cfs is similar to that with the 50-year storm flow of 600 cfs. Both profiles exhibit hydraulic jumps in the weir section near station 89 + 30 and climb from this point to about the stagnation level at the face of the bulkhead. As with all the flows, there is considerable acceleration through the transition. The drawdown under the bulkhead is very noticeable for it is about 1 ft. between the bulkhead and 10 ft. downstream.

The water surface profile for the 450 cfs flow is similar to that for the 360 cfs flow. The hydraulic jump, however, has advanced downstream to station 89 + 38. The water surface from this point to the bulkhead is nearly parallel to the pitched weir so that the distribution of the spill-over across the weir is more nearly uniform than for the other flows. The depth at the face of the bulkhead is nearly that for stagnation and, as usual, there is a marked drawdown through the bulkhead.

Figure 13 presents the profiles along the crest of weir, field edge of apron and face of downstream wing wall. The weir and apron profiles are consistent with the centerline profiles. The distribution of flow along the weir is closest to being uniform for the 115 cfs side weir discharge which occurs with a total flow of 450 cfs. The 420 cfs side weir discharge, which occurs with a total flow of 820 cfs, overtops the downstream wing wall, but the other flows are contained by the wall in its basic design position. Modifications to this wall are not studied for only the height may be changed since the length and location are fixed by field conditions. Fig. 14 shows patterns for 215 and 400 cfs flows.

The discharge curve in Fig. 8 shows that above a total inflow (Q_t) of about 300 cfs, all of the curves including the one representing the specifications show a slope ($\triangle Q_c/\triangle Q_t$) of about 0.17; however, the curves for the basic and modified No. 2 designs show respectively 25 and 10 cfs greater discharge in the lower channel than specified. In the case of Modification No. 2, the increment of 10 cfs is considered to be well within the probable error in the calculation of the storm flood flows and consequently acceptable.

Small amounts of sediment which were added to the flow at the approach were moved through the wasteway, indicating no tendency to accumulate sediment.

G. Summary of the Results of the Tests of Purisima

1. Basic Design

Unlike the Cobada wasteway the approach to Purisima is not along the centerline of the structure. Rather it is through an existing highway culvert with a slope of .0185 whose centerline makes an angle of 12° with the wasteway centerline at the culvert exit. In the basic design this turn is made abruptly without curves. This detail is shown in Fig. 3-a and discussed in Section D-2.

At the culvert exit the flow is forced to make an abrupt turn of 12° with the result that a large disturbance wave is formed off the west (right) wall. This wave is propagated downstream from one wall to the other almost all the way to the bulkhead, as is well shown for the 50-year flood flow in Fig. 15 and in the profiles in Fig. 16. The length of the wave (centerline distance between crests) is about 30 ft. for this flow and its height is about 2 ft. at station 59 + 60 (about 30 ft. downstream from the culvert exit.) Waves are also caused at the inlet to the highway culvert due to the abrupt contraction in a region where the flow velocity

is above the critical value. As can be seen by comparing the data in Figs. 6-c and 16, the waves in the Purisima flume are much higher than those in the Cobada flume, which fact accounts in part for the difference in performance between the two structures. For example, the bulkhead in Purisima did not exhibit any measurable hysteresis effect, although it has the same profile as that of the Cobada basic design, due in the main to the effect of the waves. This is discussed below in more detail.

The profiles in Fig. 16 indicate a high acceleration through the culvert and then a deceleration from the culvert exit to the upstream end of the side weir where acceleration again begins. Since there is a difference in channel width and slopes between Cobada and Purisima, it is reasonable to expect considerable difference in the condition of the flows as they approach the weir. However, the Froude numbers at a point 10 ft. upstream from the upstream edge of the weirs are 1.1 for both Cobada and Purisima when the flows are respectively 600 cfs (run 29 Cobada) and 635 cfs (run 5 Purisima).

Preliminary design computations indicated that a hydraulic jump might occur at the sharp break in grade at station 59 + 60 for a flow of 600 cfs. No jump occurred in the model. If the formula for the jump on a horizontal floor is used as an approximation, a calculation on the basis of the measured depth of 5.26 ft. at the break in grade at station 59 + 60 gives a depth downstream required for a jump as 6.9 ft. A review of the profiles in Fig. 16 will show that such a depth is not attained in the reach between the culvert exit and the upstream end of the weir. Consequently with this design, it does not appear possible for the jump to occur because the reach below the culvert exit is not long enough to permit deceleration of the flow to a depth proper for a jump. To further investigate this point, sills 1/2 ft. high were placed at and below station 59+60.

These sills did not induce a jump and, therefore, it was concluded that the jump would not occur in this reach and the study was continued on the basis of no jump between the culvert exit and weir entrance. The jump finally occurs at about midpoint in the side weir station 57 + 50.

As with the basic Cobada design, the flow over the side weir is concentrated in the downstream 30 ft. of the 70-ft. weir, as shown in Figs. 16, 17 and 18. For a 50-year storm the depth of flow at the face of the bulkhead is about 9 ft., which is about the maximum attained by the surging water. The mean depth at this point is about 8 ft., but it is difficult to measure since the flow upstream from the bulkhead is very disturbed with large up and down surges and counterclockwise eddies present at all times. The discharge under the bulkhead is contracted from a depth of 4 ft. at the bulkhead to 2.5 ft. at a point 10 ft. downstream therefrom. Across the weir crest and the field edge of the apron, the flow is concentrated in the downstream parts. As with the Cobada design, there are no provisions for energy dissipators along the field edge of the apron.

Fig. 16-a presents computed and measured profiles at the centerline of the channel for the 50-year flood flow. The calculations are based on having a hydraulic jump at station 59 + 60 and have already been discussed. The presumption of a jump at this point keeps the computed profile 2 to 3 ft. higher than that measured from station 59 + 60 to station 58 + 20, near the upstream edge of the weir. Here the disagreement decreases but the discrepancy between the calculated and measured values are still too large to be useful.

The ratio between the discharge under the bulkhead (Q_c) to the total discharge (Q_t) is measured as Q_c/Q_t = 308/635 which is in good agreement with the specified ratio of 315/635 as shown in Fig. 19. However, as mentioned before, the bulk of this flow is concentrated in the downstream 30 ft. of the weir.

At a distance of 250 ft. upstream from the culvert entrance the velocity of approach for the 50-year flow was varied between 20 and 5 fps by varying the sluice gate opening in the model at that point and keeping the flow rate constant at 635 cfs. Over this range of velocities, the same depth prevails at the culvert entrance. This indicates that the culvert acts as a control and sets up a particular ponded condition that is independent of the approach velocity.

Waves due to the abrupt change in the flow direction at the culvert exit are also present with the 10-year storm flow of 260 cfs as shown in Fig. 17-a and c, although they are not as high as with the 50-year storm. For the 10-year storm flow, the depth at the bulkhead is slightly less than 4.0 ft. and, therefore, clears the bulkhead lip which is set 4.0 ft. above the floor. The depth for a uniform flow of 260 cfs is 4.2 ft. when the slope of the 7-ft. wide rectangular channel is .00325 and its roughness factor, n, is .015. A study of the profiles in Fig. 17-a and c will show that this depth is not reached. Below station 57 + 20 the flow accelerates due to the sharp increase in slope at station 57 + 00.

The flow of 260 cfs, just discussed, passes under the bulkhead on both the rising and falling stages, so there is no measurable hysteresis effect with this design even though the bulkhead has square corners. The absence of hysteresis effect is due to the presence of large waves on the water surface which are much higher than those observed with the Cobada model. The waves keep the water surface constantly disturbed so that it is impossible for the flow to sustain for any measurable length of time the low head on the bulkhead necessary to maintain the contracted profile through the gate on the falling stage, as discussed in Section F-1. Instead, when this low head does occur, waves will tend to intermittently lower and raise the depth below and above the lip of the bulkhead. With this wavy

water surface, a cyclic make and break between the water surface and the lip of the bulkhead is observed so that the water surface upstream from the bulkhead rises and falls as though the bulkhead were "breathing." Under the same conditions the Cebada model operated smoothly. The difference between the behavior of these two structures can apparently be explained by the difference in the magnitude of the waves.

Figure 17 shows water surface profiles for flows of 405, 495, and 795 cfs with the basic design. These are similar to those with the 10- and 50-year storm flows. The profiles are characterized by convergence or acceleration through the culvert, gradual deceleration from the culvert exit to the upstream edge of the weir, acceleration through most of the side weir section, then deceleration to near the stagnation level at the face of the bulkhead. All profiles show a marked contraction through the bulkhead and then an acceleration which continues downstream to the end of the model.

As shown in Fig. 17, with the super flood of 795 cfs, the depth of water at the upstream face of the culvert is 12 ft. above the invert. Since this is at least 2 ft. higher than the Camp Cook road at this point, much of the water from this flood will spread out and spill over the road and into the fields downstream and be lost to the wasteway. This feature of the culvert approach was not modeled but it is clear that it is impossible to get a flow such as 795 cfs into the prototype wasteway. If a flow of 795 cfs should persist for any time, some water will probably flow over the road and never get back into the channel. The culvert and the road-way thus act to limit the magnitude of the flow which can enter the wasteway.

The waves along the north and south wall of the model near the culvert exit are 1 to 2 ft. high, depending on the flow, as shown in Fig.17 and, as with the 10-and 50-year storms, these waves persist in the channel

even under and downstream from the bulkhead. The super flow of 795 cfs raised the water surface to 10.2 ft. above the floor on the upstream face of the bulkhead, while with the 405 and 495 cfs flows, this depth was 6.8 and 7.7 respectively. With all the storm flows, the water surface immediately upstream from the bulkhead is especially disturbed with surges and large eddies present at all times.

The profiles along the weir crest, field edge of the apron, and face of downstream wing wall, shown in Fig. 18, are unbalanced in the downstream direction and typical of the flow distribution at these points with this design.

As shown in Fig. 19, the discharge under the bulkhead is slightly lower with the basic design than that specified, which is labeled "Comp." (for computed) in the figure. However, the difference is slight, 0 to 5 cfs, and indicates that the designers did a commendable job.

2. <u>Modification No. 1 to Basic Design of Purisima Wasteway</u>

As indicated in paragraph G-1, the Purisima channel makes a sudden 12° turn to the left at the downstream end of the road culvert. This causes waves of appreciable height which propagate downstream in the characteristic diamond pattern. Wave patterns for the Purisima channel for flows of 500, 600 and 800 cfs are shown in Fig. 20.

The problem of high velocity flow in curves has been studied by Knapp and Ippen*. Their solution to this problem involved producing a wave pattern that was 180° out of phase with the wave pattern produced in the main curve. The resulting two-wave patterns interfere with each other and result in equilibrium flow in the curve. This same idea can be applied to flow which is turned by an abrupt bend. To use this method the bends must

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*"Design of Channel Curves for High Velocity Flow," by R. T. Knapp, Proceedings ASCE, Vol. 75, No. 9, Nov. 1949, p. 1318.

be arranged in pairs such that the disturbances produced by the first bend will be cancelled by those produced at the second bend. In order to do this, the bends must have the same angle of turn and they must be located 1/2 wave length apart, where the wave length is defined as the distance measured along the center line of the channel between successive wave crests.

A drawing showing the modification of the bend in the Purisima channel according to these ideas is shown in Fig. 3-b. The distance between the bends has been determined from the data presented in Fig. 20, which shows that the wave length for flows between 400 and 500 cfs is about 25 ft. However, when the turn is reduced from 12° to 6°, the wave length will increase because the magnitude of the disturbances, and hence the wave celerity, C, will decrease. To correct for this difference, a wave length of 30 ft. was assumed and the distance between bends was made 15 ft. A study of Fig. 20-b will show that one of the turns was made 6° and the other 6° - 20'. This was necessary in order to avoid shifting the entire channel downstream from the bends. The bend design modified according to the above description is known as Modification No. 1 to the basic design.

Figures 21 and 22 present data for the 50-year storm flow in the modified bend. These indicate that the disturbance due to the bends has not been eliminated completely. However, as a comparison with Figs. 15, 16 and 17 will show, the modification effected a considerable reduction in the height of the waves in the channel and, therefore, improved the flow conditions.

Knapp and Ippen also developed a method for using diagonal sills on the bottom of a channel to reduce wave disturbances at curves. This method was also tried in the model, but it was abandoned as less desirable than making two bends.

With the modified bend and a total flow of 600 cfs, the flow under the bulkhead is 340 cfs compared to 320 cfs with the basic design, as shown in Fig. 19. The water surface profile across the side weir section for a flow of 600 cfs is lower with this modified design than with the basic design, as shown in Figs. 22 and 23; hence the head on the side weir is decreased. The flow over the side weir is distributed approximately the same as with the basic design, that is, it is concentrated along the downstream part of the side weir and over the downstream part of the apron.

With the 10-year storm of 265 cfs, Modification No. 1 decreases the waves in the channel downstream from the bends, although as with the 50-year storm, some waves are still present. It is difficult to tell how much of this wave disturbance is due to the bend and how much is due to the disturbance at the inlet end of the road culvert, since, apparently both of these sources contribute. Because of the presence of these waves, there is no measurable hysteresis effect in the flow under the bulkhead for rising and falling stages.

The structure with Modification No. 1 was studied in considerable detail for a total flow rate of 495 cfs. The water surface profiles for this flow are shown in Figure 22. In this case, the reduction in wave height seems to be greater than for the other two flows discussed above. This means that the one-half wave length of the disturbance for this flow is closer to the distance between the bends than for the other flows. Figure 22 shows that the head on the weir has been reduced by this modification and hence that the side spill also has been reduced.

3. Modification No. 2

Modification No. 2 to the basic design of the Purisima wasteway consisted of pitching the weir as was done with the Cebada wasteway to obtain a more uniform distribution of flow over the weir. This was done

by lowering the weir lip at the upstream end and raising it at the downstream end. A study of water surface profile for the 10-year flood showed that it would be possible to lower the upstream end of the weir without spilling any water at this flow. After some preliminary experiments, the weir was inclined such that its elevations at the upstream and downstream ends were, respectively, 3.3 and 6.0 ft. above the floor at these points. The bulkhead was retained as in the basic design and the apron was kept parallel to the bottom of the flume at an elevation 2.7 ft. above the channel bottom. The wing walls were kept the same as in the basic design. As shown in Fig. 3-c, the vertical face of the downstream end of the weir was made sharp rather than blunt.

Since the velocities upstream from the weir are above the critical velocity, the effects of the modification to the weir crest are noticeable only in the neighborhood of the weir section, as shown in Figs. 24, 25, 26 and 27. Figure 27 shows the water surface profiles across the weir section. From this, it can be seen that the head, and hence the flow over the weir, is more evenly distributed than it was in the basic design and that the most nearly uniform distribution of flow occurs for discharges in the neighborhood of 400 cfs. The pitch of the weir was adjusted so that the best distribution over it occurred at approximately 400 cfs total flow, since this is much more frequent occurrence than the higher flows. Photographs of discharge over the weir for total flows of 405 and 495 cfs are shown in Fig. 28.

Fig. 24-a is a confetti streak photograph of the 50-year storm flow in the Cebada wasteway. It is included here since the behavior in the two wasteways is similar. Much of the improvement in the distribution of flow over the weir can be attributed to deceleration of the flow, which was affected by increasing the height of the walls by pitching the weir.

For the 50-year flow, the wing wall at the downstream end of the weir is still considerably overtopped, but the disturbance is reduced considerably from what it was in the basic design by sharpening the vertical edge of the weir notch at the downstream end.

At the design discharge of 635 cfs, the flow under the bulkhead is 325 cfs or only 10 cfs greater than called for by the specifications, as shown in Fig. 19. This difference is considered small and tolerable.

H. Discussion of Results

The calculations for the water surface profile along the weir section and the discharge over it were made in the office of the Soil Conservation Service at Santa Barbara, California. These calculations were based on an equation which was developed by Nimmo* using the conservation of linear momentum. This reads as follows for a rectangular channel:

$$\Delta d = \frac{\Delta x \,(S - S_o) - d\,(F^2 \frac{\Delta q}{q})}{1 - F^2} \quad \ldots\ldots \quad (1)$$

where d is the depth of flow, x is the distance along centerline (positive in direction of flow), S_o and S are slope of the energy gradient and channel bottom respectively (a downward slope in the direction of flow is positive), $F = \frac{V}{\sqrt{gd}}$ (Froude number) where V is the flow velocity, g is the acceleration of gravity, q is the rate of discharge at the distance x and Δq is the discharge over the weir in the distance Δx.

When the flow velocity attains the critical value (F = 1) in the above equation, the denominator becomes zero and the value of Δd becomes infinite. This is of course physically impossible and is indicated by the equation because it was assumed in the derivation that the pressure would be hydrostatically distributed over every normal section whereas the stream

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* "Side Spillways for Regulating Diversion Canals", by W.H.R. Nimmo, Trans. A.S.C.E., 1928, Vol. 92, pp. 1561.

lines in the region of critical depth become definitely curvilinear. For this reason, the equation is not applicable when the flow is at or near critical velocities.

However, for want of a better method, the computers usedequation (1) which served well for the sub-critical velocities between the bulkhead and the point upstream where the weir drawdown curve was reached. Here the velocities are super-critical but near to critical since F, as measured varies between 1.3 and 1.7. To get the profile in this reach, the depth was alternately assumed to be sub and super critical in the calculation procedure. This gave the sawtooth profiles shown in Figs. 6 and 16, which are in marked contrast to those gradually varying ones obtained from the model study and shown in the same figures.

Through approximately the upstream two-thirds of the weir sections, a reasonable check of the low points on the calculated profiles was obtained. However, in the remaining one-third and down to the bulkhead, the measured depths were considerably higher than those calculated. This was particularly true at the bulkhead where the measured depths are always very near to the stagnation level. The calculation of the water depth, a short distance upstream from the bulkhead, cannot be done with the ordinary equation for varied flow because of the curvature and vertical accelerations in this region. The depth, d, at the bulkhead may be expressed by the relation for the rate of discharge, q, through a sluice gate,

$$q = C \text{ (area) } \sqrt{2\,g\,(d - \frac{a}{2})}$$

where a is the height of the opening, c is the discharge coefficient, and g is the acceleration of gravity. Values of C obtained from measurements in the Cobada model are included in the appendix of this report.

The weir causes a particular drawdown curve near to its upstream end and in this reach the velocities are super critical. When a bulkhead

is placed downstream from the weir so as to obstruct the flow, the downstream depths rise to very near stagnation level at the bulkhead and the flow backs upstream. Since the backwater from the bulkhead is at subcritical velocity, a hydraulic jump must occur at some point upstream between the backwater from the bulkhead and the weir drawdown curve.

Because a jump occurs in the weir section, it is not possible to calculate the surface profile in this reach by means of equation (1). This explains the rather erratic results obtained by the calculations which are shown on Figs. 6 and 16. Incidentally, it was the erratic nature of those results that convinced the designers of the need for laboratory study of this problem.

The behavior in the upstream section of the weir is affected by cross waves similar to those which occur at the angle bend at the culvert exit in the Purisima channel. The disturbance in this section is caused by the discontinuity in the side wall as the flow enters the weir section. At this point, the side support is suddenly removed from the flow and a negative wave or disturbance is propagated diagonally across the stream. This type of disturbance is of the same kind that occurs on the inside of the angle bend in the Purisima channel. This disturbance may be seen clearly in Figs. 11-b and 24-b. The abrupt drawdown starting from the upstream end of the weir notch and extending diagonally across the channel in the downstream direction is caused by this negative wave. The fact that these waves exist makes this a three-dimensional problem which cannot be solved by the simplified theory expressed by equation 1. The solution of this problem could be obtained by the methods outlined by Ippen* for analyzing high-velocity flow.

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* "Mechanics of Supercritical Flow," Proceedings of ASCE, Vol. 75, No. 9, Nov. 1949, p. 1290, (by A. T. Ippen)

I. Sediment Problems

In order for the channel system to function, the concrete channels must be able to transport all of the sediment coming into them from the earth channels. The problem of transporting the sediment in the lined channels is made more difficult because at high flow an appreciable part of the total flow is diverted without diverting very much of the sediment. This occurs because the diversion at the wasteway structures is performed by skimming water from the surface where the sediment content is less than in the lower portions of the flows.

An analysis of the problem may be made by using one of the sediment transportation formulas, for instance the Duboys formula*,

$$G = b \psi T (T - T_c) \qquad (1)$$

where G is the rate of sediment transportation, b is the channel width, ψ is a factor determined by the sediment, T is the shear stress at the bed and T_c is the critical value of the shear stress at which the sediment will commence to move. If the lined channel is to operate without filling up with sediment, its capacity, G_2, to transport sediment must equal or exceed the capacity, G_1, of the earth channel or:

$$G_2 \geqq G_1 \qquad (2)$$

Substituting (1) into (2) and noting the ψ is the same in the earthen and lined channel, we get

$$b_2 T_2 (T_2 - T_c) \geqq b_1 T_1 (T_1 - T_c) \qquad (3)$$

The shear stress T is given by

$$T = WRS \qquad (4)$$

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* "Fluid Mechanics for Hydraulic Engineers" by Hunter Rouse, McGraw-Hill, 1938, p. 335.

where W is the specifed weight of water, R is the hydraulic radius of the section and S is the slope of the channel. If T_c is small compared with T_1 and T_2, equation 3 can be simplified to read:

$$b_2 T_2^2 \geqq b_1 T_1^2 \qquad (5)$$

The quantity bT^2 is proportional to the sediment transporting capacity of the channel as long as T_c is small compared with T and, therefore, equation 5 states the condition that has to be fulfilled if the lined channel is to scour clean. The value of T can be calculated from equation 4 by using R calculated from the Chezy and Manning formulas.

Einstein* has shown that if the banks of a stream are stabilized, such as with vegetation, the value of R to be used in equation 4 is smaller than that ordinarily used in the velocity formulas and has developed a method for calculating it. The equations are:

$$\frac{1.486}{n_b} R_b^{2/3} S^{1/2} = \frac{1.486}{n} R_w^{2/3} S^{1/2} \qquad (6)$$

$$R_b P_b + R_w P_w = A \qquad (7)$$

where the subscripts "b" and "w" refer to the bed and walls or banks of the channel, respectively, so that n_b is the roughness of the bed proper and n_w is the roughness of the banks and P_b and P_w are, respectively, the wetted perimeter of the bed and the wetted perimeter of the banks.

The value of n_w is estimated in the usual manner. Einstein calculates n_b from the Strickler formula and then applies a correction which is a function of the rate of sediment transportation. In the present analysis n_b was estimated to be .015 in both earthen channels. This value is larger than given by the Strickler formula and allows for some increase in roughness due to bed irregularities that may develop.

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* "Formulas for the Transportation of Bed Load," by H. A. Einstein, Trans. A.S.C.E., 1942

When the banks are not stabilized, the value of R and, hence, T is larger than with stabilized banks. Therefore, with unstabilized banks the value of bT^2 and the rate of sediment transportation are also larger.

The properties of the earthen and concrete channel of the Cebada and Purisima systems in the immediate vicinity of the wasteway structuros are tabulated below:

	Cebada Channel	Purisima Channol
(1) Earthen Channel		
bottom width (b)	10'	10'
side slopes	1 1/2:1	1:1
Bottom Slope (S)	.005	.007
Average sediment size	0.25 mm	.15 mm
(n_w) Roughness of banks	.035	.035
(n_b) Roughness of bed	0.015	.015
(2) Concrete Channol		
width (b)	8 ft.	6.5'
bottom slope (S)	.009	.0109
$n = n_b = n_w$	.015	.015

The function bT^2 for the two earthen and concrete soctions is plotted on Fig. 28-1 as a function of total discharge. It will be noted that thore aro two curves for each earthen channel. In one of these (labeled bed and walls) the value of R in equation 4 has been calculated for unstabilized banks assuming n = .035. In the other (labeled bed only) stabilized banks were assumed and n_w and n_b were taken or .035 and .015, respectively, and R_b calculated from equations 6 and 7 was used in equation 4 to calculate T.

In these calcuations it became evident that it was permissible to assume that T_c was small compared with T. Values of T_c for sediment

of the sizes present in these channels* is in the neighborhood of .020 lbs. per sq. ft. while T is of the order of 1 1/2 lbs. per sq.ft. which is over 50 times T_c.

In Fig. 28-1 the values of bT^2 for the earthen and the concrete channels may be compared directly. It is seen that the sediment transporting capacity of the Purisima earthen channel with nonstabilized banks is about twice that of the concrete channel. When stable banks are assumed the two capacities are about equal. The capacity of the earthen channel in the Cobada stream is less than that of the concrete channel if the banks are assumed to be stabilized, but for unstabilized banks and high flows the capacity of the earthen channel exceeds that of the concrete channel.

Attention is called to the rapid falling off of the quantity bT^2 for the concrete channels when the total discharge exceeds about 300 cfs. This occurs because for the higher flows much of the water is spilled out of the channel at the wasteway structures so that although the total discharge continues to increase, the flow in the concrete channel increases very little.

The significance of the results shown on Fig. 28-1 can be determined only after a careful review of the calculations and the relations on which they are based. To begin with, it is assumed that the Duboys formula applies to these channels. This formula applies well to streams where the rate of sediment transportation is relatively low and the amount of material in suspension is small or negligible. In the Purisima and Cobada streams, the sediment transportation rates are high and much of the material must move in suspension so the applicability of the formula is not certain. In this analysis it was assumed that the grades of the

*H. R. Doc. No.238, 73rd Cong. 2nd Session, Feb. 5, 1934, Appendix XV.

earth channel upstream from the lined sections would not change. However, the grade could steepen appreciably if material was brought into this section at a rate that was higher than its present transporting capacity. Such a condition could be brought about by lack of control in the system upstream from this point. Furthermore, there are such complicating factors as the culvert at the inlet to the Purisima wasteway which must also be considered in evaluating these results. From these comments it is seen that the results of the calculations can be taken only as a qualitative indication of the performance of the channel. Even though this is the case, the results are still useful. They show that the transporting capacity of the concrete channels tends to reach a limit after side spill starts, while that of the earth channel tends to increase continuously. The importance of installing and maintaining suitable stabilizing works cannot be over emphasized, for the success of the entire system depends to a large extent upon the proper performance of this part of the work. This means that the critical condition for clogging the concrete channels with sediment will occur at the high flows when side spill is going on. It appears from the analysis that the concrete channels have enough capacity to carry the sediment load but that there is no great margin of safety. If bank caving in the upper regions of the stream were allowed to go on and the stream were to be too heavily loaded, there is danger that the concrete channels would fill up with sediment and overflow. This means that it is extremely important to stabilize the earth channels with dams and bank protection in order to insure satisfactory functioning of the system.

J. Conclusions

1. Cebada Wasteway

(a) The design as proposed does not give the desired division of flow between the lined channel and side weir. In addition the spill is concentrated in the downstream 30 percent of the side weir.

(b) The 10-year storm flow will be fully contained in the concrete channel if the lip of the bulkhead is set 3.4 ft. above the floor at station 88 + 68 and if the side weir is pitched uniformly from 4.2 ft. off the floor at its upstream end to 5.6 ft. off the floor at its downstream end. To obtain proper distribution of the 50-year storm flow the lip of the bulkhead must be rounded in the form of a circular curve of 1/2 ft. radius tangent to the upstream face of the bulkhead as shown in Fig. 2-c. This increases the coefficient of discharge.

(c) A very noticeable hysteresis effect persists with the 10-year storm flow and a bulkhead with right angle corners as in the basic design. Rounding the upstream lip of the bulkhead decreases the effect but does not eliminate it entirely.

(d) Uniform distribution of discharge does not prevail over the 70 ft. long side weir for the entire range of flows associated with the 10 through 50 year storms even when the side weir is pitched. The best conditions are when the discharge over the weir is 80 cfs with a total inflow of 400 cfs.

(e) The contraction of the channel from a trapezoidal to rectangular cross-section through the transition section causes disturbance waves to be formed in the channel below the transition. While these have a magnitude of approximately 1/4 ft. with a discharge of 600 cfs, they do not interfere with the operation of the structure.

2. Purisima Wastoway

(a) The design as proposed permits an acceptable division between the total inflow and the discharge under the bulkhead. However, the discharge over the side weir is concentrated in the downstream 30 percent of the weir length for most of the flows.

(b) Modification of the basic design such that the side weir is pitched uniformly from an elevation of 3.3 ft. above the floor at the upstream end to 6.0 ft. above the floor at the downstream end improves the distribution of the flow over the side weir. This modification gives the most nearly uniform flow distribution when the total inflow is 350 cfs.

(c) The design as proposed will induce the formation of unnecessarily high disturbance waves at the exit from the culvert where the flow is turned abruptly through an angle of 12° to the left (looking downstream). These waves persist throughout the full length of the rectangular channel to the bulkhead. They effect the discharge over the side weir and necessitate high side walls to hold the flows.

(d) Modification of the basic design at the culvert exit by using two angle bends such that two interfering disturbance waves are set up causes a decrease in the amplitude of the waves downstream.

(e) The discharge under the bulkhead for the 10-year storm is the same on the rising as on the falling stage. The bulkhead exhibits no measurable hysteresis effect as did that for the Cebada Wasteway.

(f) A hydraulic jump at station 59 + 60, as predicted in the design calculations, does not occur in the model, nor is it induced by the action of blocks or sills. Because of this, the measured model depths are much lower than those calculated in the stretch from the culvert to the weir section even with the large disturbance waves.

(g) The flow in the channel and in the weir section is not affected by variation in the velocity of approach to the culvert. The culvert acts as a control for the flow below it. The drawdown through the culvert is as predicted in the basic design calculation.

3. <u>General</u>

(a) The side weirs with a square cross-section perform satisfactorily. The distribution across them is not uniform but generally the nappe springs clear of the weir.

(b) The downstream wing walls are overtopped considerably by the spill from the 50-year frequency storm flowsand to a lesser extent by the spill from all flows in excess of 400 cfs.

(c) The upstream wing walls are unnecessary as a protection against flows over the side weir. The depth of flow along the face of these walls does not exceed .1 ft. even for the extreme flow of 830 cfs.

(d) The depths of flow along the field edge of the apron are considerable for the design flows. The approximate maximum velocity is 7 fps when the total flow over the apron is 300 cfs with the 50-year storm flow. This flow will erode the field downstream from the apron an amount which will vary with the type of cover present since artificial control structures are not planned.

(e) The water level at the upstream face of the bulkheads rises very nearly to the stagnation level for all flows.

(f) A sharp instead of a blunt edge for the vertical downstream faces of the side weirs reduces the heights of the disturbances at these points.

(g) An approximate analysis of the sediment carrying capacities of the concrete and earth channels in the Cobada and Purisima watersheds indicates that the concrete channels have sufficient capacity to keep scoured clean and prevent sediment from depositing and filling them up provided that the earth channels are stabilized to prevent excessive loads of sediment from being picked up and brought into the concrete channel system.

K. Recommendations

1. Cebada Wasteway

(a) It is recommended that the side weir be pitched uniformly such that the upstream end is 4.2 ft. above the floor and the downstream end is 5.6 ft. above the floor. The weir is to be 1/2 ft. thick and flat crested with 90° corners as shown in Fig. 2.

(b) It is recommended that the bulkhead be set 3.4 ft. above the floor at station 88 + 68 and that it be 1/2 ft. thick and that the upstream lip of the bulkhead be rounded by a circular curve of 1/2 ft. radius tangent to the upstream face of the bulkhead and perpendicular to the downstream face.

(c) It is recommended that the side walls from the bulkhead to station 88 + 78 be raised to a height of at least 9 ft. The walls in the other stretches of the flume may be lowered on the basis of the profiles contained in Figs. 6 and 12.

(d) It is recommended that the wing wall at the downstream end of the weir be raised to at least 5 ft. above the apron floor.

2. Purisima Wasteway

(a) It is recommended that the alignment of the channel from the culvert exit, station 59 + 95 be as shown in Fig. 3-b, i.e., with a 6° left turn looking downstream at station 59 + 95, a 6° - 20' left turn at station 59 + 80 and a 20' right turn at station 57 + 00.

(b) It is recommended that the side weir be pitched uniformly such that the upstream end is 3.3 ft. above the floor and the downstream end is 6.0 ft. above the floor, as shown in Fig. 3-c. The weir is to be 1/2 ft. wide, and flat crested with 90° corners as in the basic design.

(c) It is recommended that the bulkhead be set vertically 4.0 ft. above the floor at station 57 + 20 and that it be 1/2 ft. thick and have 90° corners.

(d) It is recommended that the side walls between the bulkhead and station 57 + 30 upstream be raised to a height of at least 10 ft. The walls in the other stretches of the flume studied may be lowered on the basis of the profiles in Figs. 16 and 25.

3. General

(a) It is recommended that a continuing program of maintenance for the erosion control structures and the channel in the Purisima and Cebada Watersheds be set up.

(b) It is recommended that the growing of erosion resistant cover crops be encouraged downstream from the aprons to assist in protecting the land from erosion during the storm flows of over a 10 year frequency.

APPENDIX

A. Results of Tests of Several Bulkheads

During the course of the model tests it was necessary to determine the discharge coefficient for several bulkheads since for such devices data are not readily available in the pertinent literature. Figure 29 shows three types of bulkheads which were tested. All the model bulkheads were constructed of plywood and had two coats of ordinary enamel applied to them. They felt smooth to the touch. The dimensions shown in Fig. 29 are those for the Cebada prototype for which the linear scale ratio was 1 model equals 12 prototype. The tests were conducted using the model scale with a channel bottom slope of .003 in all cases.

The coefficient of discharge, C, was calculated from the expression,

$$q = C\, a\, b \sqrt{2g\,(d - a/2)}$$

where q is the discharge under the bulkhead, a is the distance from the bottom of the channel to the lip of the bulkhead, b is the width of the channel, g is the acceleration of gravity and d is the water depth at the face of the bulkhead.

Table A-1 below presents the data on the bulkheads shown in Fig. 29.

Table A-1

Average Values of the Coefficient of Discharge for Three Bulkheads

	Type of Bulkhead		
Head (ft)	A	B	C
4	.61	.68	.76
5	.59	.67	.75
6	.57	.66	.74
7	.55	.65	.73
8	.53	.64	.72

A coefficient of .58 was used in the design computations for a flat, square cornered bulkhead for a calculated depth (d) of 5.1 ft. so the experimentally determined coefficient as presented in Table A-1 is in excellent agreement with this value.

However the coefficients in Table A-1 show a decrease with an increase in head which is in a direction opposite to those shown in King* for gates tested under similar conditions.

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* "Handbook of Hydraulics" by H. W. King, 1929, pp. 75.